CHAPTER 3

SOIL PARAMETERS

3-1. **Methodology.** A site investigation and soil exploration program of the proposed construction area should be initially completed to obtain data required for analysis of bearing capacity. Estimates of ultimate and allowable bearing capacity using analytical equations that model the shear failure of the structure along slip surfaces in the soil and methods for analyzing in situ test results that model the bearing pressures of the full size structure in the soil may then be carried out as described in Chapter 4 for shallow foundations and Chapter 5 for deep foundations. The scope of the analysis depends on the magnitude of the project and on how critical the bearing capacity of the soil is to the performance of the structure.

a. **Soil Parameters.** The natural variability of soil profiles requires realistic assessment of soil parameters by soil exploration and testing. Soil parameters required for analysis of bearing capacity are shear strength, depth to groundwater or the pore water pressure profile, and the distribution of total vertical overburden pressure with depth. The shear strength parameters required are the undrained shear strength C_u of cohesive soils, the effective angle of internal friction ϕ' for cohesionless soils, and the effective cohesion c' and angle of internal friction ϕ' for mixed soils that exhibit both cohesion and friction.

b. **Use of Judgment.** Judgment is required to characterize the foundation soils into one or a few layers with idealized parameters. The potential for long-term consolidation and settlement must be determined, especially where soft, compressible soil layers are present beneath the foundation. Assumptions made by the designer may significantly influence recommendations of the foundation design.

c. **Acceptability of Analysis.** Acceptability of the bearing pressures applied to the foundation soil is usually judged by factors of safety applied to the ultimate bearing capacity and estimates made of potential settlement for the bearing pressures allowed on the foundation soil. The dimensions of the foundation or footing may subsequently be adjusted if required.

3-2. **Site Investigation.** Initially, the behavior of existing structures supported on similar soil in the same locality should be determined as well as the applied bearing pressures. These findings should be incorporated, using judgment, into the foundation design. A detailed subsurface exploration including disturbed and undisturbed samples for laboratory strength tests should then be carried out. Bearing capacity estimates may also be made from results of in situ soil tests. Refer to EM 1110-1-1804 for further information on site investigations.

a. **Examination of Existing Records.** A study of available service records and, where practical, a field inspection of structures supported by similar foundations in the bearing soil will furnish a valuable guide to probable bearing capacities.

(1) **Local Building Codes.** Local building codes may give presumptive allowable bearing pressures based on past experience. This information should only be used to supplement the findings of in situ tests and analyses using one or more

methods discussed subsequently because actual field conditions, and hence bearing pressures, are rarely identical with those conditions used to determine the presumptive allowable bearing pressures.

(2) **Soil Exploration Records.** Existing records of previous site investigations near the proposed construction area should be examined to determine the general subsurface condition including the types of soils likely to be present, probable depths to groundwater level and changes in groundwater level, shear strength parameters, and compressibility characteristics.

b. **Site Characteristics.** The proposed construction site should be examined for plasticity and fissures of surface soils, type of vegetation, and drainage pattern.

(1) **Desiccation Cracking.** Numerous desiccation cracks, fissures, and even slickensides can develop in plastic, expansive soils within the depth subject to seasonal moisture changes, the active zone depth Z_a, due to the volume change that occurs during repeated cycles of wetting and drying (desiccation). These volume changes can cause foundation movements that control the foundation design.

(2) **Vegetation.** Vegetation desiccates the foundation soil from transpiration through leaves. Heavy vegetation such as trees and shrubs can desiccate foundation soil to substantial depths exceeding 50 or 60 ft. Removal of substantial vegetation in the proposed construction area may lead to significantly higher water tables after construction is complete and may influence bearing capacity.

(3) **Drainage.** The ground surface should be sloped to provide adequate runoff of surface and rainwater from the construction area to promote trafficability and to minimize future changes in ground moisture and soil strength. Minimum slope should be 1 percent.

(4) **Performance of Adjacent Structures.** Distortion and cracking patterns in nearby structures indicate soil deformation and the possible presence of expansive or collapsible soils.

c. **In Situ Soil Tests.** In the absence of laboratory shear strength tests, soil strength parameters required for bearing capacity analysis may be estimated from results of in situ tests using empirical correlation factors. Empirical correlation factors should be verified by comparing estimated values with shear strengths determined from laboratory tests. The effective angle of internal friction ϕ' of cohesionless soil is frequently estimated from field test results because of difficulty in obtaining undisturbed cohesionless soil samples for laboratory soil tests.

(1) **Relative Density and Gradation.** Relative density and gradation can be used to estimate the friction angle of cohesionless soils, Table 3-1a. Relative density is a measure of how dense a sand is compared with its maximum density.

TABLE 3-1

Angle of Internal Friction of Sands, ϕ'

a. Relative Density and Gradation
(Data from Schmertmann 1978)

Relative Density D_r, Percent	Fine Grained		Medium Grained		Coarse Grained	
	Uniform	Well-graded	Uniform	Well-graded	Uniform	Well-graded
40	34	36	36	38	38	41
60	36	38	38	41	41	43
80	39	41	41	43	43	44
100	42	43	43	44	44	46

b. Relative Density and In Situ Soil Tests

Soil Type	Relative Density D_r, Percent	Standard Penetration Resistance N_{60} (Terzaghi and Peck 1967)	Cone Penetration Resistance q_c, ksf (Meyerhof 1974)	Friction Angle ϕ', deg		
				Meyerhof (1974)	Peck, Hanson and Thornburn (1974)	Meyerhof (1974)
Very Loose	< 20	< 4	----	< 30	< 29	< 30
Loose	20 - 40	4 - 10	0 - 100	30 - 35	29 - 30	30 - 35
Medium	40 - 60	10 - 30	100 - 300	35 - 38	30 - 36	35 - 40
Dense	60 - 80	30 - 50	300 - 500	38 - 41	36 - 41	40 - 45
Very Dense	> 80	> 50	500 - 800	41 - 44	> 41	> 45

(a) ASTM D 653 defines relative density as the ratio of the difference in void ratio of a cohesionless soil in the loosest state at any given void ratio to the difference between the void ratios in the loosest and in the densest states. A very loose sand has a relative density of 0 percent and 100 percent in the densest possible state. Extremely loose honeycombed sands may have a negative relative density.

(b) Relative density may be calculated using standard test methods ASTM D 4254 and the void ratio of the in situ cohesionless soil,

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}} \cdot 100 \qquad (3\text{-}1a)$$

$$e = \frac{G}{\gamma_d} \cdot \gamma_w - 1 \qquad (3\text{-}1b)$$

where

e_{min} = reference void ratio of a soil at the maximum density
e_{max} = reference void ratio of a soil at the minimum density
G = specific gravity
γ_d = dry density, kips/ft^3
γ_w = unit weight of water, 0.0625 kip/ft^3

The specific gravity of the mineral solids may be determined using standard test method ASTM D 854. The dry density of soils that may be excavated can be determined in situ using standard test method ASTM D 1556.

(2) **Standard Penetration Test (SPT).** The standard penetration resistance value N_{SPT}, often referred to as the blowcount, is frequently used to estimate the relative density of cohesionless soil. N_{SPT} is the number of blows required to drive a standard splitspoon sampler (1.42" I.D., 2.00" O.D.) 1 ft. The split spoon sampler is driven by a 140-lb hammer falling 30 inches. The sampler is driven 18 inches and blows counted for the last 12 inches. N_{SPT} may be determined using standard method ASTM D 1586.

(a) The N_{SPT} value may be normalized to an effective energy delivered to the drill rod at 60 percent of theoretical free-fall energy

$$N_{60} = C_{ER} \cdot C_N \cdot N_{SPT} \tag{3-2}$$

where

N_{60} = penetration resistance normalized to an effective energy delivered to the drill rod at 60 percent of theoretical free-fall energy, blows/ft
C_{ER} = rod energy correction factor, Table 3-2a
C_N = overburden correction factor, Table 3-2b

N_{SPT} may have an effective energy delivered to the drill rod 50 to 55 percent of theoretical free fall energy.

(b) Table 3-1 illustrates some relative density and N_{60} correlations with the angle of internal friction. Relative density may also be related with N_{60} through Table 3-2c.

(c) The relative density of sands may be estimated from the N_{spt} by (Data from Gibbs and Holtz 1957)

$$D_r \approx 100\left(\frac{N_{SPT}}{12\sigma'_{vo}+17}\right)^{0.5} \tag{3-3a}$$

where D_r is in percent and σ'_{vo} is the effective vertical overburden pressure, ksf.

(d) The relative density of sands may also be estimated from N_{60} by (Jamiolkowski et al. 1988, Skempton 1986)

$$D_r \approx 100\left(\frac{N_{60}}{60}\right)^{0.5} \tag{3-3b}$$

where $D_r \geq 35$ percent. N_{60} should be multiplied by 0.92 for coarse sands and 1.08 for fine sands.

(e) The undrained shear strength C_u in ksf may be estimated (Bowles 1988)

$$C_u \approx 0.12 N_{SPT} \tag{3-4}$$

TABLE 3-2

Relative Density and N_{60}

a. Rod Energy Correction Factor C_{ER}
(Data from Tokimatsu and Seed 1987)

Country	Hammer	Hammer Release	C_{ER}
Japan	Donut	Free-Fall	1.3
	Donut	Rope and Pulley with special throw release	1.12*
USA	Safety	Rope and Pulley	1.00*
	Donut	Rope and Pulley	0.75
Europe	Donut	Free-Fall	1.00*
China	Donut	Free-Fall	1.00*
	Donut	Rope and Pulley	0.83

*Methods used in USA

b. Correction Factor C_N (Data from Tokimatsu and Seed 1984)

C_N	σ'_{vo}*, ksf
1.6	0.6
1.3	1.0
1.0	2.0
0.7	4.0
0.55	6.0
0.50	8.0

*σ'_{vo} = effective overburden pressure

c. Relative Density versus N_{60}
(Data from Jamiolkowski et al. 1988)

Sand	D_r , Percent	N_{60}
Very Loose	0 - 15	0 - 3
Loose	15 - 35	3 - 8
Medium	35 - 65	8 - 25
Dense	65 - 85	25 - 42
Very Dense	85 - 100	42 - 58

(3) **Cone penetration test (CPT).** The CPT may be used to estimate both relative density of cohesionless soil and undrained strength of cohesive soils through empirical correlations. The CPT is especially suitable for sands and preferable to the SPT. The CPT may be performed using ASTM D 3441.

(a) The relative density of several different sands can be estimated by (Jamiolkowski et al. 1988)

$$D_r = -74 + 66 \cdot \log_{10} \frac{q_c}{(\sigma'_{vo})^{0.5}} \tag{3-5}$$

where the cone penetration resistance q_c and effective vertical overburden pressure σ'_{vo} are in units of ksf. The effective angle of internal friction ϕ' can be estimated from D_r using Table 3-1a. Table 3-1b provides a direct correlation of q_c with ϕ'.

(b) The effective angle of internal friction decreases with increasing σ'_{vo} for a given q_c as approximately shown in Figure 3-1. Increasing confining pressure reduces ϕ' for a given q_c because the Mohr-Coulomb shear strengh envelope is nonlinear and has a smaller slope with increasing confining pressure.

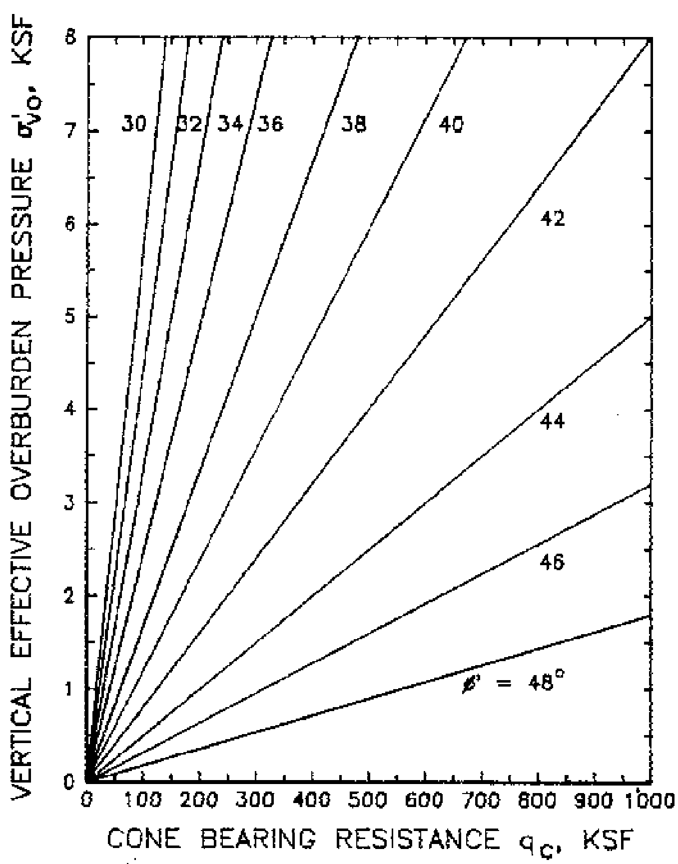

Figure 3-1. Approximate correlation between cone penetration resistance, peak effective friction angle and vertical effective overburden pressure for uncemented quartz sand (After Robertson and Campanella 1983)

(c) The undrained strength C_u of cohesive soils can be estimated from (Schmertmann 1978)

$$C_u = \frac{q_c - \sigma_{vo}}{N_k} \tag{3-6}$$

where C_u, q_c, and the total vertical overburden pressure σ_{vo} are in ksf units. The cone factor N_k should be determined using comparisons of C_u from laboratory undrained strength tests with the corresponding value of q_c obtained from the CPT. Equation 3-6 is useful to determine the distribution of undrained strength with depth when only a few laboratory undrained strength tests have been performed. N_k often varies from 14 to 20.

(4) **Dilatometer Test (DMT).** The DMT can be used to estimate the overconsolidation ratio (OCR) distribution in the foundation soil. The OCR can be used in estimating the undrained strength. The OCR is estimated from the horizontal stress index K_D by (Baldi et al 1986; Jamiolkowski et al 1988)

$$OCR = (0.5K_D)^{1.56} \quad if\ I_D \le 1.2 \tag{3-7a}$$

$$K_D = \frac{p_o - u_w}{\sigma'_{vo}} \tag{3-7b}$$

$$I_D = \frac{p_1 - p_o}{p_1 - u_w} \tag{3-7c}$$

where

p_o = internal pressure causing lift-off of the dilatometeter membrane, ksf
u_w = in situ hydrostatic pore pressure, ksf
p_1 = internal pressure required to expand the central point of the dilatometer membrane by ≈ 1.1 millimeters
K_D = horizontal stress index
I_D = material deposit index

The OCR typically varies from 1 to 3 for lightly overconsolidated soil and 6 to 8 for heavily overconsolidated soil.

(5) **Pressuremeter Test (PMT).** The PMT can be used to estimate the undrained strength and the OCR. The PMT may be performed using ASTM D 4719.

(a) The limit pressure p_L estimated from the PMT can be used to estimate the undrained strength by (Mair and Wood 1987)

$$C_u = \frac{p_L - \sigma_{ho}}{N_p} \tag{3-8a}$$

$$N_p = 1 + \ln \frac{G_s}{C_u} \tag{3-8b}$$

where

p_L = pressuremeter limit pressure, ksf
σ_{ho} = total horizontal in situ stress, ksf
G_s = shear modulus, ksf

p_L, σ_{ho}, and G_s are found from results of the PMT. Equation 3-8b requires an estimate of the shear strength to solve for N_p. N_p may be initially estimated as some integer value from 3 to 8 such as 6. The undrained strength is then determined from Equation 3-8a and the result substituted into Equation 3-8b. One or two iterations should be sufficient to evaluate C_u.

(b) σ_{ho} can be used to estimate the OCR from $\sigma'_{ho}/\sigma'_{vo}$ if the pore water pressure and total vertical pressure distribution with depth are known or estimated.

(6) **Field Vane Shear Test (FVT).** The FVT is commonly used to estimate the in situ undrained shear strength C_u of soft to firm cohesive soils. This test should be used with other tests when evaluating the soil shear strength. The test may be performed by hand or may be completed using sophisticated equipment. Details of the test are provided in ASTM D 2573.

(a) The undrained shear strength C_u in ksf units is

$$C_u = \frac{T_v}{K_v} \tag{3-9}$$

where

T_v = vane torque, kips·ft
K_v = constant depending on the dimensions and shape of the vane, ft^3

(b) The constant K_v may be estimated for a rectangular vane causing a cylinder in a cohesive soil of uniform shear strength by

$$K_v = \frac{\pi}{1728} \cdot \frac{d_v^2 h_v}{2} \cdot \left[1 + \frac{d_v}{3h_v}\right] \tag{3-10a}$$

where

d_v = measured diameter of the vane, in.
h_v = heasured height of the vane, in.

K_v for a tapered vane is

$$K_v = \frac{1}{1728} \cdot [\pi d_v^3 + 0.37(2d_v^3 - d_r^3)] \tag{3-10b}$$

where d_r is the rod diameter, in.

(c) Anisotropy can significantly influence the torque measured by the vane.

d. **Water Table.** Depth to the water table and pore water pressure distributions should be known to determine the influence of soil weight and surcharge on the bearing capacity as discussed in 1-4d, Chapter 1.

(1) **Evaluation of Groundwater Table (GWT).** The GWT may be estimated in sands, silty sands, and sandy silts by measuring the depth to the water level in an augered hole at the time of boring and 24 hours thereafter. A 3/8 or 1/2 inch diameter plastic tube may be inserted in the hole for long-term measurements. Accurate measurements of the water table and pore water pressure distribution may be determined from piezometers placed at different depths. Placement depth should be within twice the proposed width of the foundation.

(2) **Fluctuations in GWT.** Large seasonal fluctuations in GWT can adversely influence bearing capacity. Rising water tables reduce the effective stress in cohesionless soil and reduce the ultimate bearing capacity calculated using Equation 1-1.

3-3. **Soil Exploration.** Soil classification and index tests such as Atterberg Limit, gradations, and water content should be performed on disturbed soil and results plotted as a function of depth to characterize the types of soil in the profile. The distribution of shear strength with depth and the lateral variation of shear strength across the construction site should be determined from laboratory strength tests on undisturbed boring samples. Soil classifications and strengths may be checked and correlated with results of in situ tests. Refer to EM 1110-2-1907 and EM 1110-1-1804 for further information.

a. **Lateral Distribution of Field Tests.** Soil sampling, test pits, and in situ tests should be performed at different locations on the proposed site that may be most suitable for construction of the structure.

(1) **Accessibility.** Accessibility of equipment to the construction site and obstacles in the construction area should be considered. It is not unusual to shift the location of the proposed structure on the construction site during soil exploration and design to accommodate features revealed by soil exploration and to achieve the functional requirements of the structure.

(2) **Location of Borings.** Optimum locations for soil exploration may be near the center, edges, and corners of the proposed structure. A sufficient number of borings should be performed within the areas of proposed construction for laboratory tests to define shear strength parameters C_u and ϕ of each soil layer and any significant lateral variation in soil strength parameters for bearing capacity analysis and consolidation and compressibility characteristics for settlement analysis. These boring holes may also be used to measure water table depths and pore pressures for determination of effective stresses required in bearing capacity analysis.

(a) Preliminary exploration should require two or three borings within each of several potential building locations. Air photos and geological conditions assist in determining location and spacings of borings along the alignment of proposed levees. Initial spacings usually vary from 200 to 1000 ft along the alignment of levees.

(b) Detailed exploration depends on the results of the preliminary exploration. Eight to ten test borings within the proposed building area for typical structures are often required. Large and complex facilities may require more borings to properly define subsurface soil parameters. Refer to TM 5-818-1 for further information on soil exploration for buildings and EM 1110-2-1913 for levees.

b. **Depth of Soil Exploration.** The depth of exploration depends on the size and type of the proposed structure and should be sufficient to assure that the soil supporting the foundation has adequate bearing capacity. Borings should penetrate all deposits which are unsuitable for foundation purposes such as unconsolidated fill, peat, loose sands, and soft or compressible clays.

(1) **10 Percent Rule.** The depth of soil exploration for at least one test boring should be at the depth where the increase in vertical stress caused by the structure is equal to 10 percent of the initial effective vertical overburden stress beneath the foundation, Figure 3-2. Critical depth for bearing capacity analysis D_c should be at least twice the minimum width of shallow square foundations or at least 4 times the minimum width of infinitely long footings or embankments. The depth of additional borings may be less if soil exploration in the immediate vicinity or the general stratigraphy of the area indicate that the proposed bearing strata have adequate thickness or are underlain by stronger formations.

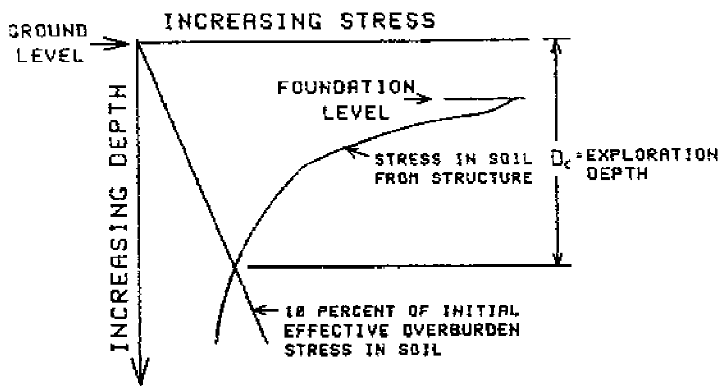

Figure 3-2. Estimation of the critical depth of soil exploration

(2) **Depth to Primary Formation.** Depth of exploration need not exceed the depth of the primary formation where rock or soil of exceptional bearing capacity is located.

(a) If the foundation is to be in soil or rock of exceptional bearing capacity, then at least one boring (or rock core) should be extended 10 or 20 ft into the stratum of exceptional bearing capacity to assure that bedrock and not boulders have been encountered.

(b) For a building foundation carried to rock 3 to 5 rock corings are usually required to determine whether piles or drilled shafts should be used. The percent recovery and rock quality designation (RQD) value should be determined for each rock core. Drilled shafts are often preferred in stiff bearing soil and rock of good quality.

(3) **Selection of Foundation Depth.** The type of foundation, whether shallow or deep, and the depth of undercutting for an embankment depends on the depths to acceptable bearing strata as well as on the type of structure to be supported.

(a) Dense sands and gravels and firm to stiff clays with low potential for volume change provide the best bearing strata for foundations.

(b) Standard penetration resistance values from the SPT and cone resistance from the CPT should be determined at a number of different lateral locations within the construction site. These tests should be performed to depths of about twice the minimum width of the proposed foundation.

(c) Minimum depth requirements should be determined by such factors as depth of frost action, potential scour and erosion, settlement limitations, and bearing capacity.

c. **Selection of Shear Strength Parameters.** Test data such as undrained shear strength C_u for cohesive soils and the effective angle of internal friction ϕ' for cohesionless sands and gravels should be plotted as a function of depth to determine the distribution of shear strength in the soil. Measurements or estimates of undrained shear strength of cohesive soils C_u are usually characteristic of the worst temporal case in which pore pressures build up in impervious foundation soil immediately following placement of structural loads. Soil consolidates with time under the applied foundation loads causing C_u to increase. Bearing capacity therefore increases with time.

(1) **Evaluation from Laboratory Tests.** Undrained triaxial tests should be performed on specimens from undisturbed samples whenever possible to estimate strength parameters. The confining stresses of cohesive soils should be similar to that which will occur near potential failure planes in situ.

(a) Effective stress parameters c', ϕ' may be evaluated from consolidated-undrained triaxial strength tests with pore pressure measurements (R) performed on undisturbed specimens according to EM 1110-2-1906. These specimens must be saturated.

(b) The undrained shear strength C_u of cohesive foundation soils may be estimated from results of unconsolidated-undrained (Q) triaxial tests according to EM 1110-2-1906 or standard test method ASTM D 2850. These tests should be performed on undrained undisturbed cohesive soil specimens at isotropic confining pressure similar to the total overburden pressure of the soil. Specimens should be taken from the center of undisturbed samples.

(2) **Estimates from Correlations.** Strength parameters may be estimated by correlations with other data such as relative density, OCR, or the maximum past pressure.

(a) The effective friction angle ϕ' of cohesionless soil may be estimated from in situ tests as described in section 3-2c.

(b) The distribution of undrained shear strength of cohesive soils may be roughly estimated from maximum past pressure soil data using the procedure outlined in Table 3-3. Pressure contributed by the foundation and structure are not included in this table, which increases conservatism of the shear strengths and avoids unnecessary complication of this approximate analysis. σ_{vo} refers to the total vertical pressure in the soil excluding pressure from any structural loads. σ'_{vo} is the effective vertical pressure found by subtracting the pore water pressure.

TABLE 3-3

Estimating Shear Strength of Soil From Maximum Past Pressure
(Refer to Figure 3-3)

Step	Description
1	Estimate the distribution of total vertical soil overburden pressure σ_{vo} with depth and make a plot as illustrated in Figure 3-3a.
2	Estimate depth to groundwater table and plot the distribution of pore water pressure γ_w with depth, Figure 3-3a.
3	Subtract pore water pressure distribution from the σ_{vo} distribution to determine the effective vertical soil pressure distribution σ'_{vo} and plot with depth, Figure 3-3a.
4	Determine the maximum past pressure σ'_p from results of laboratory consolidation tests, in situ pressuremeter or other tests and plot with depth, Figure 3-3b.
5	Calculate the overconsolidation ratio (OCR), σ'_p/σ'_{vo}, and plot with depth, Figure 3-3c.
6	Estimate C_u/σ'_{vo} from $$\frac{C_u}{\sigma'_{vo}} = 0.25(OCR)^{0.8} \qquad (3\text{-}11)$$ where C_u = undrained shear strength and plot with depth, Figure 3-3c.
7	Calculate C_u by multiplying the ratio C_u/σ'_{vo} by σ'_{vo} and plot with depth, Figure 3-3d.
8	An alternative approximation is $C_u \approx 0.2\sigma'_p$. For normally consolidated soils, $C_u/\sigma'_p = 0.11 + 0.0037 \cdot PI$ where PI is the plasticity index, percent (Terzaghi and Peck 1967)

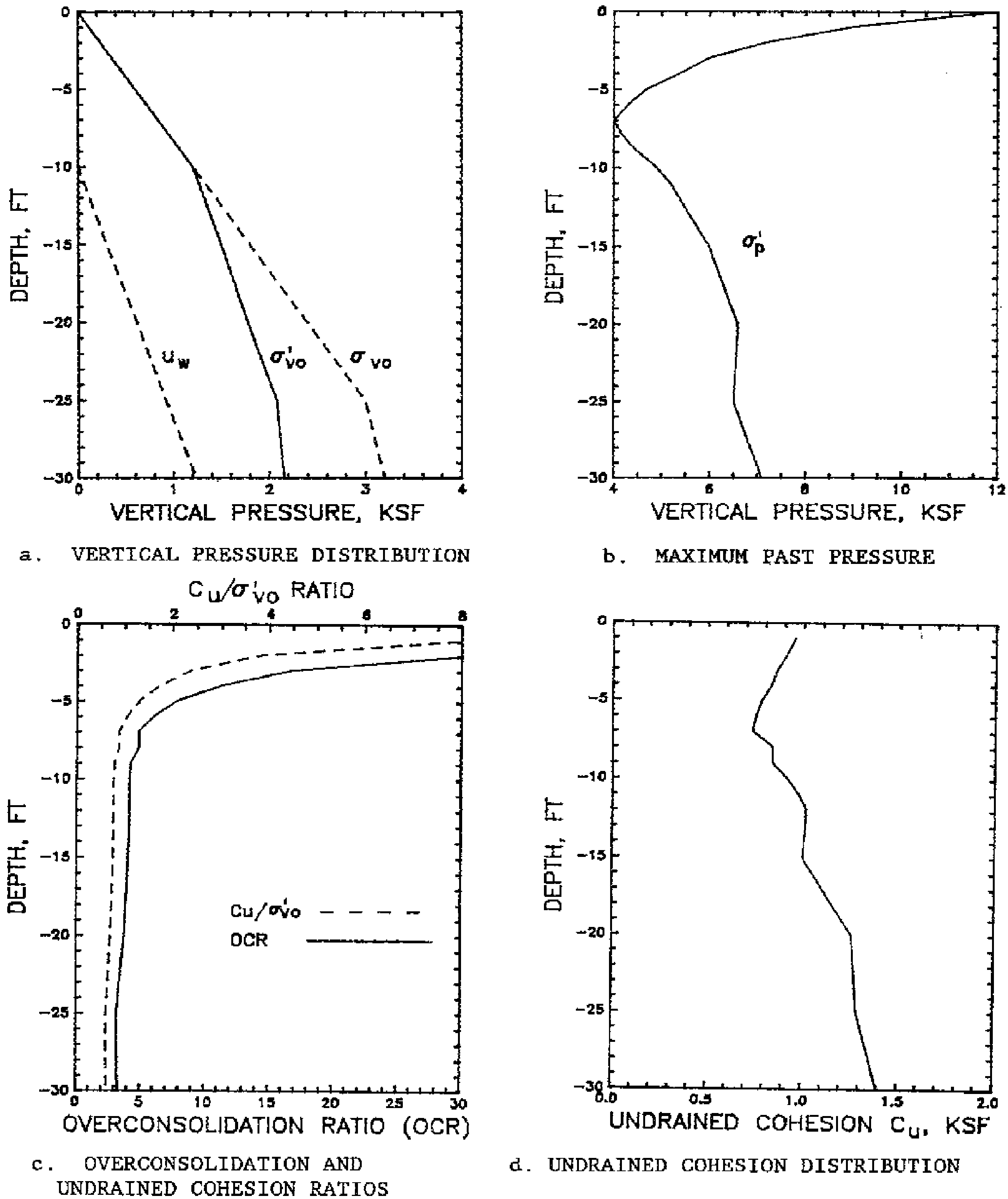

Figure 3-3. Example estimation of undrained strength
from maximum past pressure data